POTENTIAL CAUSES OF HISTORICAL LANDSLIDE AND A PROBABILISTIC APPROACH TO ASSESSING FUTURE RISK FOR DEVELOPMENT OF JEROME REST AREA

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Abstract: The site of the proposed Jerome Rest Area is located near the head scarp of an historic landslide in the town of Jerome, Arizona. Based on the results of geotechnical and geological analyses, the landslide on which the rest area would be located is considered dormant at present. Factors that could cause landslide recurrence were evaluated in a limited risk analyses. The limited risk analysis for landslide stability incorporated identifiable geotechnical parameters, but not all external factors, that could lead to landslide recurrence.

Possible causes of original landslide movement in 1936 are: low shear strength near surface soils; high groundwater conditions; leaking water and fire pipelines; surface water concentration near the head scarp; breaks in the concrete ditch on Cleopatra Hill immediately above the slide area; assimilated seismic events created by Coyote blasts at United Verde Mine; mine blasts from the UVX Mine located practically beneath the landslide; movement along the Verde fault from the Coyote blasts or a seismic event; oversteepening of some slopes to construct buildings; and soil creep. Possible causes to remobilize the landslide are: low shear strength in the near surface soils; high groundwater conditions; a seismic event that could contribute to re-mobilizing creep; movement along the Verde fault from a seismic event; and failure of lower portions of the slide area caused by recent fill placement and the potential of progressive failures up hill in the existing landslide mass.

A cross-section through the landslide area and the portions of the slope above and below the site of the rest area was developed. A specific failure plane was analyzed based on data obtained from the historical review and field explorations. Slope stability analyses models were analyzed for 1936 conditions and for 2002 conditions.

The landslide is considered stable under 2002 conditions provided there are no strong ground motion forces added to the slope and based on current groundwater conditions. The results of the stability analyses indicate the safety factor for instability is highly sensitive to the magnitude of potential seismic events.

Due to the uncertainties associated with any geotechnical investigation and the factors that can cause instability, all slopes, even those with factors of safety greater than 1.0, have some potential for failure. The higher the computed factor of safety is for a given slope, the lower its probability of failure. Assessing the probability of failure through a probabilistic analysis, along with a limited risk analysis was performed for this study.

Geotechnical parameters were varied from their respective average by a percentage of their standard deviation using a Monte Carlo probabilistic approach. Results of the probabilistic slope stability analyses show there is generally a one in seven (15%) chance of slope instability under 2002 conditions when the seismic coefficient is 0.02g. When the seismic coefficient is 0.10g the probability is generally one in two (55%).

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PROJECT INFORMATION

ADOT is planning the construction of the Jerome Rest Area to be located in the Lower Park parking area on the west side of Hull Avenue opposite the Sliding Jail Park in Jerome, Arizona. The proposed structure will be one story in height facing Hull Avenue. In addition to the building structure, Lower Park parking lot improvements and sidewalks along Hull Avenue are planned.

SITE CONDITIONS

Surface Conditions: The site of the proposed rest area is an existing gravel parking lot for Lower Park in Jerome, Arizona. There is an existing landscaped slope from Main Street down to the west side of the parking lot. Although the head scarp for the landslide was located further west near the middle of Main Street, the slope generally depicts the extent of the head scarp for the 1936 landslide.

The existing site conditions of the area in and around the 1936 landslide generally consisted of: the buildings west of Main Street, Main Street, the slope from Main Street down to Lower Park, Lower Park and associated parking lot, a portion of Hull Avenue, the newly constructed dry block retaining wall, the Sliding Jail Park and associated parking lot and basketball court, recently placed fill below the Sliding Jail Park and parking lot, and the existing ground surface downslope of the recently placed fill.

The majority of the buildings along the west side of Main Street show evidence of cracks that have been repaired at sometime in the past. Concrete retaining walls at various locations behind these buildings showed cracking at the face of the wall. Otherwise, the buildings and retaining wall structures appear to be in relatively good condition considering their age.

The ground surface downslope of the recently placed fill is hummocky and irregular in appearance near what appears to be the central portion of the old landslide. The undulations are on the order of one to 5 feet in height. Vegetation in this area is thick with grasses, cattails, bushes, and trees. The ground surface was mushy and soft in places indicating groundwater apparently comes to the surface in these areas. Along the flanks of the landslide the ground surface appears to have been bermed.

SCOPE OF SERVICES

The scope of the services performed for this study included the following:

- Site reconnaissance by Terracon in collaboration with ADOT personnel;
- Site reconnaissance by a third party geologist; Mr. Paul Lindberg;
- Subsurface exploration including installation of two slope inclinometers with coaxial cable attached for time domain reflectometry (TDR) readings;
- Laboratory testing including moisture content, sieve analyses with hydrometer, and plasticity index;
- Historical document review; and,
- Engineering analyses including a limited risk analyses.

The borings were drilled to approximate depths of 87 to 100 feet. Selected soil samples were tested for moisture content, sieve analyses with hydrometer, and plasticity index.

HISTORICAL DOCUMENT REVIEW

As a part of our scope of services, a limited risk analyses with regard to constructing the rest area on what is known to be a recent (1936) landslide was requested. To better define parameters for the risk analyses, an understanding of the cause(s) of the 1936 landslide was considered essential. The historical documents provided by ADOT for review of the potential cause(s) of the landslide were comprehensive, but not exhaustive. The following summaries and abstracts have been provided with an emphasis on understanding site conditions, external influences and subsurface conditions that may have caused the 1936 landslide.

Historical Summaries

The Town of Jerome

Jerome is situated on the northwest slope of the Black Hills above the Verde Valley. The Town exists because of two large-scale copper mining operations. There were (and still are) many mines in the area, but the two largest, the United Verde Mine and the United Verde Extension Mine, were by far the most significant. Separate Historical Summaries for these mines are presented below.

United Verde Mine – 1888 to 1953

1888 (Canty, 1987) William Andrews Clark, ex-Senator from Montana bought up outstanding stock and restarted operations. The mine was an underground operation. A two-furnace smelter operated in the gulch above Jerome at the current location of the open pit. Meanwhile, exploratory mining revealed rich ore bodies.

1920 (Alenius, 1930) Stripping for open-pit mining begins. Drilling and blasting was necessary to reduce the massive rock materials for excavation. Early in the stripping operation, conventional methods were adequate. At the 160-foot level, the blasting crews resorted to “coyote hole” blasting in unaltered diorite.

Coyote Hole Blasting: This method relies on adits with powder pockets at their distal end. The adits and pockets were excavated using conventional blasting. The 4½x6-foot adits were 30 feet apart and extended horizontally, into the bench to be blasted, a distance between 78 and 135 feet. The 5x5-foot powder pockets extend downward, approximately 30 feet, to the base of the desired final slope. Several *tons* of explosives, including black powder and dynamite were placed in each pocket. (Imagine a 5x5-foot shaft, 30 feet deep, filled to nearly half full with explosives.) The blasts comprised six to fourteen coyote holes and six blasts were used during stripping. The mine engineers also used several techniques to deal with “hot holes,” which occurred where mine fire heat and humidity could affect the blasting.
Major blasts occurred in: 1) December, 1924; 2) March, 1925; 3) December, 1925; 4) March 6, 1926; and November 23, 1926 (The listing is incomplete. The dates of the other two major blasts are unknown.).

1931 (Canty, 1987) Operations ceased. The Great Depression had started and copper prices sagged.

1935 (Canty, 1987) Phelps Dodge buys out the heirs of William Clark. Mining operations resume and continue through the Second World War and into the 1950s.

1953 Phelps Dodge shuts down the mine.

United Verde Extension Mine – 1912 to 1935

1912 (Canty, 1987) James S. Douglas (Rawhide Jimmy) organized the United Verde Extension Company.

1915 Production mining begins. Mining methods were limited to the square-set, stope filled method. (Timber framed structures are advanced through the stopes so the timbers provide roof support. Once stopes are removed, the voids are filled with waste material and then the pillars between stopes are removed. It was vital that crushing of pillars or portions of ore bodies be prevented because the friction heat could ignite a mine fire in the sulphurous material.)

1935 (Canty, 1987) Still owned by Douglas, the mine – played out – is shut down.

The Major Landslide in Jerome – 1924 to 1939

Periodically, landslides have damaged homes, retaining walls, and other structures (e.g. concrete ditches) throughout the Town since the earliest days. The buried utilities along the Town's roads were an ongoing maintenance issue. Significant movements affecting the business center of the Town became an issue in the 1920s:

1924 The first noticeable movements on Main and Hull streets. (Note that the earliest known date of Coyote blasting was December of 1924.)

1926 The Episcopal Church "became unstable …moved three feet off its base." This building was demolished and replaced with what is currently the History Center. (There is no discussion of the relative locations of the demolished structure verses the History Center.)

1927 The south wing of the United Verde Clubhouse had to be destroyed. This structure was originally built as the third United Verde Hospital. It sits directly on the Verde Fault.

1934 Strong ground movement along the Verde fault caused cracks in the old United Verde Hospital and the retaining wall below it.

In September of 1935, an engineer working for UVX, Mr. J. William Waara, began monitoring subsidence and ground movements in a large area of the business district of Jerome. The area that he studied extended from School Gulch all the way to the end of Main Street. Subsidence in an area encompassing much of the business district was calculated using survey data from road surveys. Subsidence at a curb on Clark Street west of the Clubhouse (on the footwall side of the Verde Fault) was two hundredths of an inch. Along Hull Avenue, Main Street, and Clark Street to the north, subsidence varied from 1.2 to 2.0 feet. In September of 1936, while the subsidence was being studied, movement in an area centered along Conglomerate Street from Main Street down to Rich Street, accelerated and became a landslide. Interestingly, as the landslide movements began, the subsidence movements measured over the larger area of the Town essentially stopped.

Landslide: The landslide was centered along Conglomerate Street and extended from Main down to Rich Street, which was between Hull Street and Juarez Street. The slide affected structures in the area between the Grand Central Hotel and the Bartlett Hotel. The landslide never slumped as a rapid failure, instead, the surface moved downslope over the course of months. As the movements accumulated, buildings were damaged and ultimately, many buildings were demolished. The old Jail building, essentially a concrete box, moved downslope some 225 feet. Of the affected buildings, only the Jail still stands.

As the landslide moved, Mr. Waara studied the geometry of the moving mass as well as groundwater conditions. He collected information from several shafts that were excavated through the moving mass, below groundwater, past failure planes, and into underlying conglomerate "bedrock". The shafts were located at several locations in and around the landslide. Mr. Waara was able to measure the depth of soil layers as well as the depth to groundwater, failure planes, and "bedrock". He was also able to make detailed observations of movements at shear planes. Based on his observations, the depth to groundwater varied from 10 to 20 feet below the surface. The failure planes were observed at depths typically from 25 to 40 feet below the surface, although a few were shallower.

He also attempted to quantify the subsurface groundwater in the landslide by a study during heavy rains. He went further and excavated pits (the Tisdale pits) hoping to find the anticipated volume of groundwater flowing in the subsurface near the toe of the landslide. The volume of water observed did not match his estimate, but he was able to prove that significant groundwater was present and flowing in the landslide mass.

A Mr. John Quigley worked as the Town Engineer and also studied subsidence and landslides in the town. He also inventoried damage to municipal facilities and his calculations and estimates were the basis for a claim by the Town against UVX. A partial accounting of damages yielded the widely published value of $134,871.16 in damages. The various landowners were also filing claims.

Arbitration: In 1938, the management of the two major mining companies (Louis Cates of Phelps Dodge, owner of the United Verde Mine and Jim Douglas, President of United Verde Extension Mine), agreed that the property owners and the Town should not be ruined by the landslide. They did not, however, agree as to their relative responsibility. In 1938, they decided to hire an arbitrator.

They engaged the services of Mr. Ira S. Joralemon, a geologist and recognized expert. He spent weeks listening to testimony from experts for both mines and assembling information. He then spent some weeks in his office in San Francisco distilling the information and generating a chart. At his request, the expert witnesses and management from both mines met and reviewed the chart. Subsequently, they visited the section of the Hopewell Tunnel where it intersects the Verde fault. Using flashlights, he was able to show an offset of the timber bents of the roof supports extending away from the fault zone. This was conclusive proof of fault movement contemporary to blasting for the United Verde open pit mine and the land movements in Jerome. Mr. Joralemon decided that UVX was responsible for 2/3^{rds} of the damages and United Verde/Phelps Dodge was responsible for 1/3rd. The available documentation implies that the arbitrated decision was used to settle claims by building owners as well as the Town.

The available documentation does not clearly present the data and analyses showing that ground movements in Jerome were the result of mine subsidence caused by UVX although that was clearly the starting point for Joralemon's decision. It seems likely that he had access to the survey data developed by Mr. Waara and attributed the observed elevation changes to mine subsidence.

Small verses UVX: The Small Building was a large retail structure fronting Main Street. While not clearly documented (in the information available to us), it appears that the structure was located at the corner of 1st Street and Main, opposite the Bartlett Hotel, which still stands. The Small Building was not destroyed by the landslides of 1936. In the years that followed, however, the building experienced significant movements and distress. Mr. Small sued UVX and in 1939, the case went to trial. The available documentation includes notes taken by an observer during the testimony of several witnesses and a report submitted by Mr. Waara. The building was demolished within a few years of the trial.

During the trial, Mr. Waara expressed his opinion during the testimony that mine subsidence was not a contributory factor to initiating the landslide. He did not deny that subsidence had occurred. In fact, map exhibits (not available in the documents provided to us) provided with his report showed contours of subsidence. He testified that the subsidence was over a large area and even at the edges of the subsidence zone, differential settlements that would affect structures was small enough to prevent significant distress to buildings. Based on his experience and investigation, he believed that groundwater had "lubricated" a slippage plane and increased the driving forces (by saturating the mass on top of the failure plane) to instigate the landslide. He explained that there were three other landslides in the area that occurred with no mines beneath them and no towns above them so there was only a one in four chance that the human activities caused the slide.

Synopsis of Historical Conditions: Landslides are noted to exist in the vicinity of Jerome and the Verde fault as far back as 1898. The UVX Mine began operations in 1915. Open pit mining began at the United Verde mine in 1920 with Coyote blasting techniques employed between 1924 and 1927 to speed removal of overburden materials. Movement in Main Street and Hull Avenue are first noticed in 1924. Two structures located near the Verde fault are demolished in 1926 and 1927 due to movement of the structures. The United Verde Mine ceased operations in 1931 due in part to the Great Depression. In 1935 the United Verde Mine resumes operations under Phelps Dodge and the UVX Mine shuts down having exhausted the ore body. In September of 1936 the rate of movement of the landslide increased significantly. Many buildings were destroyed or demolished, and underground utilities broken due to the landslide. Of the buildings within the landslide, only the "Sliding Jail" survived after movement had ceased in 1940.

SUBSURFACE CONDITIONS

Geology: The project area is located in the transition zone physiographic province (Cooley, 1967) of the North American Cordillera (Stern, et al, 1979) of Arizona, which separates the high plateau to the north from the basin and range topography of southern Arizona. Geologic conditions in the area of the landslide "has undergone dramatic geologic changes over the past 75 million years" (Lindberg, 2002). "Precambrian age Cleopatra Rhyolite is exposed for more

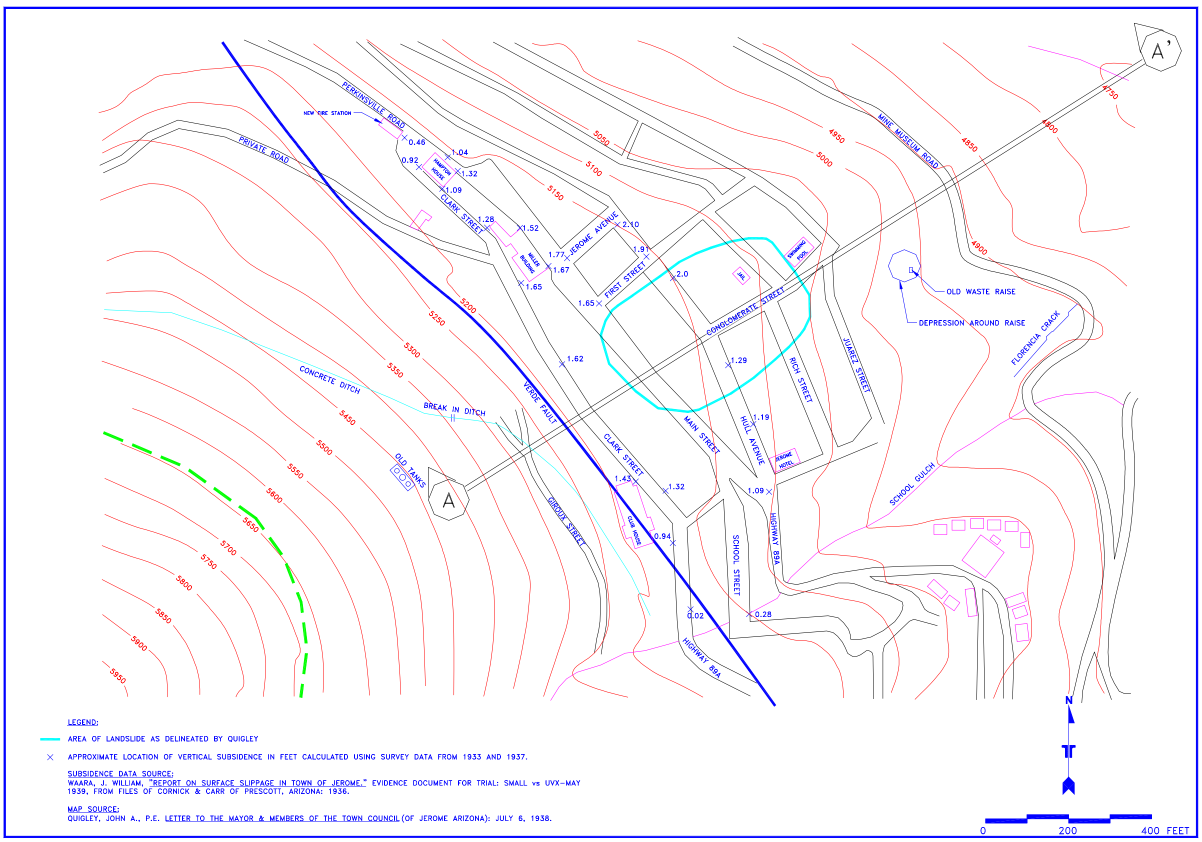

FIGURE 1: Quigley Map of Jerome in 1936 depicting the extent of the landslide.

than 800 feet in elevation directly above the plane of the Verde fault that passes through the upper part of the Jerome townsite. Tertiary age Hickey Basalt, dated at 10-15 million years old, has been dropped approximately 1550 feet against the Precambrian basement rocks along the Verde fault. Most of the town of Jerome is situated on top of surface colluvium and Hickey Basalt bedrock."

"The Verde fault plane has an attitude of about –60° to the northeast and lies well below the landslide area. The most recent period of faulting took place approximately 8 million years ago. The ancestral phase of the Verde fault, however, experienced a period of high angle reverse motion during the Laramide Uplift that occurred ~75 million years ago. During that time the northeastern side of the fault was raised several hundred feet higher than the southwestern side. The Ancestral Verde fault plane was reactivated ~8 million years ago when it experienced a normal drop to the east-northeast. As a result, the rocks within the Verde fault zone have been severely crushed and subjected to deep weathering over the past 75 million years. Jerome area underground and drill hole results show that flexures in the Verde fault zone can vary form a few inches to 30 or more feet wide." Figure 3 shows the present day geologic cross section through the town of Jerome.

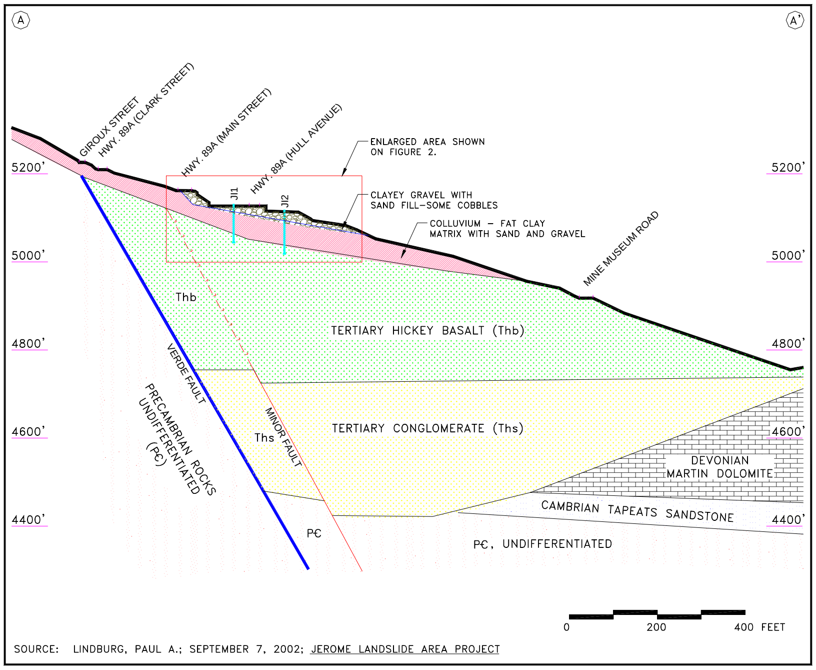

FIGURE 2: Cross section through landslide depicting geology and surface geometry.

Geomorphology: It is the opinion of Terracon that the landslide site and surrounding land both up and down hill of the site indicate there has been previous mass movement by earthflows or landslides. This is most readily observed by the "chute" shape of the ground.

Site and Regional Seismicity: The Verde fault is a northwest trending, basin and range normal fault that separates the Black Hills (Mingus Mountain) from the Verde Valley (Lam et al, 1992). The maximum credible earthquake was estimated to be in the range of Richter Magnitude 7.0+. There is a 90 percent probability of non-exceedance in 50 years of a seismic event with horizontal ground movement of magnitude 0.14g in the Jerome area.

The dominant faults in the area are the Verde and Coyote faults, which are northwest-trending normal faults (Sanchez, 1987). The Verde fault zone has many parallel and sub-parallel faults within several miles of its location. The majority of fault movement occurred during the Pleistocene age. Younger movement on the fault has probably occurred, but can not be proven. Two historic earthquakes have been documented to have occurred in 1931 and 1985 with magnitudes between 3.0 and 5.0. The maximum probable earthquake was given a magnitude

6.0 with a recurrence period of 50 years. The maximum horizontal ground acceleration was given to range between 0.18g to 0.36g.

Soil and Bedrock Conditions: The subsurface soils encountered in the test borings generally consisted of clayey gravel fill materials overlying clay rich colluvium soils overlying the Tertiary Hickey Basalt. The fill materials ranged in depth from 16 to 28 feet. The colluvium soils generally extended to depths of 65 to 84 feet (or ranged in thickness from 50 to 55 feet). The borings were terminated at penetrations of 16 to 25 feet in the Tertiary Hickey Basalt. The upper gravel materials generally consisted of mine slag or Cleopatra Rhyolite. The clay soils generally have very high plasticity indices ranging up to 56.

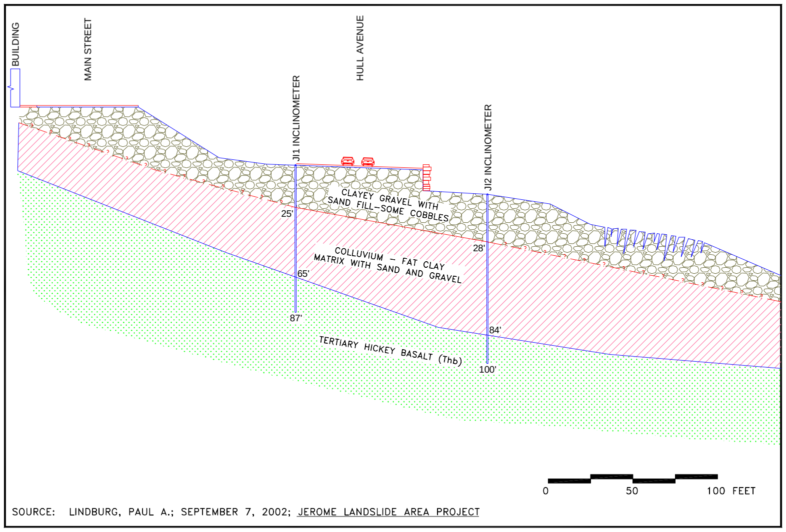

FIGURE 3: Close up of cross section shown in Figure 2.

Field Test Results: Field penetration test results for Boring No. JI1 indicate the fill materials are generally poorly compacted. The underlying soils from 16 to 24½ feet vary from loose to medium dense in relative density. The fat clay soils below a depth of 24½ feet are generally hard in consistency. Field penetration test results for Boring No. JI2 indicate the fill materials are generally well compacted, however, the penetration tests appear to represent the presence of gravel and cobbles, and not the surrounding matrix. The fat clay soils at a depth of 35 feet are generally soft to medium stiff in consistency. The clay and sand soils generally increase in consistency and relative density below a depth of 40 feet. The basalt bedrock in both borings varies from highly weathered to slightly weathered.

Inclinometer Measurement Results: Based on conversations with ADOT personnel ground movement has not been measured in either inclinometer since their installations on September 11 and 17, 2002. ADOT has acquired data on at least two subsequent visits to the site.

Time Domain Reflectometry Results: Field measurement data was obtained from TDR cables installed at Boring Nos. JI1a and JI2a on October 23, 2002 by Kane Geo Tech, Inc. The results of the TDR data indicated the slope had not moved between installation of the TDR cables and October 23, 2002. A typical graph of the TDR data is presented below.

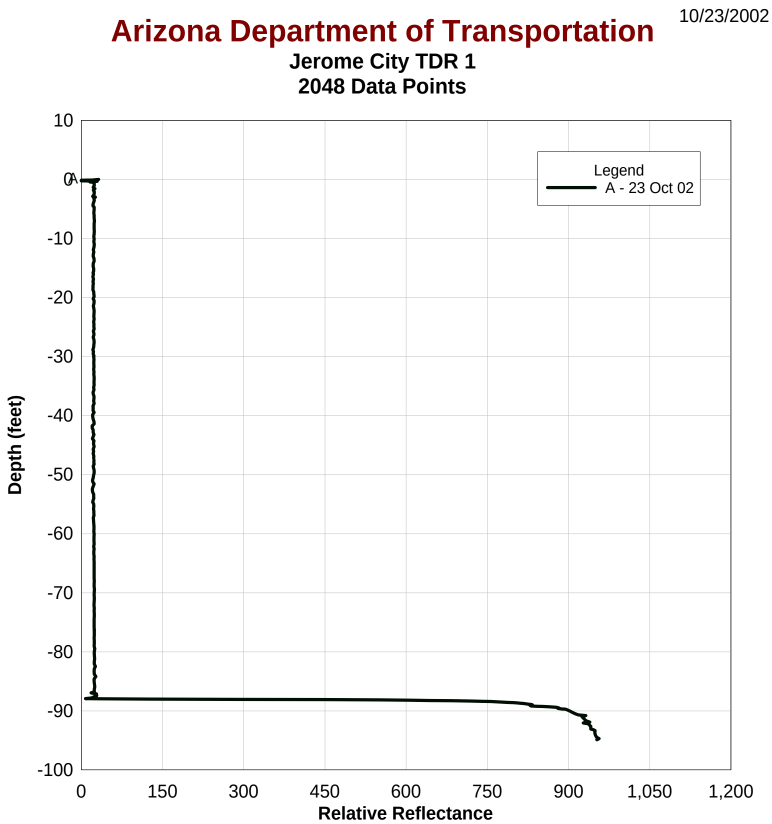

FIGURE 4: TDR plot for J1a.

Laboratory Test Results: Laboratory test results indicate the moisture content of the subsurface soils varies from 6 to 23 percent. The moisture content of the upper 25 feet of materials in the test borings is generally low. The moisture contents of the materials below depths of 25 feet are relatively high, ranging between 15 and 23 percent. The clay fraction ($<2\mu$) ranges from approximately 10 to 50 percent. The liquid limit and plasticity index range from 32 to 77, and 16 to 56, respectively.

Groundwater Conditions: Groundwater conditions were not specifically evaluated during drilling operations due to coring methods used for advancement of the borings and due to flushing the borings with water at the time of installation of the inclinometer casing.

Historically, groundwater during the 1936 slide was measured at depths on the order of 10 to 15 feet below the ground surface, according to measurements taken by Waara from 1935 to 1936.

ENGINEERING ANALYSES

Landslide Characterization: The aerial extent of the landslide is approximately 3 acres and extends generally from Main Street to the east approximately 500 feet based on the historical documents. The failure plane(s) within the landslide vary in depth from 6 to 35 feet based on the historical documents, and from 16 to 28 feet based on our field exploration. Based on this geometry and the general characterization of the landslide movement, the 1936 landslide is characterized as a slab translational slide. The depth to length ratio is less than 0.1, which, according to Skempton (1953) would be the upper limit for slab translational slides. These types of failures primarily occur in weathered clay or in shallow slope debris, and the failure surface is nearly parallel to the ground surface.

Measurements of the sliding jail and historical records indicate the landslide moved 225 feet over a period of 5 years. However, during this same period of time there is no specific evidence recorded that a large toe bulge had been created. This suggests the near surface materials were sliding/moving over soils at greater depth causing multiple failure planes as witnessed by Waara. This failure mechanism would be consistent with the minor amounts of surface disturbance observed downhill of the Sliding Jail Park parking area.

Mine Subsidence: Though Joralemon concluded mine subsidence was the major contributor to the cause of the mass movements observed with the 1936 landslide, it is the opinion of Terracon that mine subsidence had little, if any effect. It is conspicuous that Joralemon comes to his conclusion by only talking to mine experts, reviewing Quigley's information and on the basis of his own observations. There appears to have been no subsurface information gathered regarding the immediate soils or groundwater conditions at the time of his arbitration. It is also conspicuous that Joralemon concludes that UVX is $2/3^{rds}$ responsible and United Verde is $1/3^{rd}$ responsible when in fact for the prior 5 years, Joralemon had performed United Verde's tax value assessments.

Considering the depth of the ore body below the landslide site was on the order of 1100 feet, and the presence of at least 200 to 300 feet of Hickey Basalt was between the area of major mine workings and the ground surface, it is our opinion that arching in the bedrock would have easily supported 100 feet of overburden soils, and thus could have precluded any surface expression of mine subsidence.

Causes of Landslide and Potential Causes for Recurrence: The cause of the 1936 landslide is not specifically known, nor is the determination of the cause the purpose of this study. Joralemon believed the cause was related to mine subsidence. Waara explained he thought the landslide had been caused by high groundwater, and was not specifically related to mine subsidence.

Based on our analyses of historic conditions, site geology and geotechnical conditions, it is the opinion of Terracon that the possible causes of landslide movement in 1936, include the following:

- Low shear strength soils in the near surface allowing for the development of failure planes at shallow depths;
- High groundwater conditions caused by heavy rainfall events, leaking water and fire pipelines, surface water concentration near the head scarp, and breaks in the concrete ditch on Cleopatra Hill immediately above the slide area;
- Assimilated seismic events created by Coyote blasts at the United Verde Mine may have initiated the mass movement of the landslide into a state of creep. In

addition, the mine blasts from the UVX Mine located practically beneath the landslide may have helped to keep the landslide in a state of continual creep;

- A seismic event in 1931 may have contributed to the creep movement of the landslide area;
- Movement along the Verde fault from the Coyote blasts or the seismic event, and a subsequent potential for change in the groundwater regime due to the offset in the Verde fault located up gradient of the site;
- Oversteepening of some slopes to construct buildings (such as on the fill sides of Main and Hull) leading to minor earth movement during the time when the ground relaxed into an active lateral earth pressure state;
- Soil creep is considered to be a contributing factor to the 1936 slide since historical data indicates that minor earth movement had been noted all around town since the time of the Coyote blasts and even before. The ground may have begun to creep in the mid 1920's and continued to creep until the remaining factors came together to cause significant mass movement and the landslide in 1936.

At present, it appears the landslide is not in an active state of creep. Continued measurements of the inclinometers and TDR are being performed to confirm this assumption.

Strength Parameters: Strength parameters for geotechnical analyses could not be established by direct laboratory testing because of poor sample quality. For purposes of the engineering analyses, published correlations were used to estimate strength parameters based on available laboratory test results.

- **Residual Cohesion:** The values used for the residual cohesion of the soils along the estimated failure plane of the landslide were approximated using a correlation based on moisture content and index properties (Fang, 1991). The anticipated variability of residual cohesion of the soil value was approximated using the variability of the liquid limit test data. For our analyses, an average residual cohesion value of 100 psf was used. The variability was approximated using a standard deviation of 25 psf.

- **Effective Residual Friction Angle:** Six different correlations were used to estimate the effective residual friction angle of the colluvium soils. Data from all of the samples from Boring Nos. JI1 and JI2 were included in the data set.

  The average values and values at two standard deviations away from the average were used with the correlation charts to estimate the range of effective residual friction angles expected based on the laboratory data from the two borings.

  For our analyses, an effective residual friction angle of 16 degrees was used together with a standard deviation value of four (4) degrees.

Slope Stability Analyses: Stability analyses were performed using the computer program SLOPE/W version 5.11 developed by GEO-SLOPE International, Ltd. SLOPE/W utilizes algorithms to solve the Morgenstern-Price limit equilibrium method of slices. This method satisfies force equilibrium in both the horizontal and vertical planes and also satisfies moment equilibrium. Direction of the resultant inter-slice forces is determined using an arbitrary function. The percentage of the function, λ, required to satisfy moment and force equilibrium is computed with a rapid solver.

For purposes of our stability evaluations, a cross-section (A - A') through the landslide area and the portions of the slope above and below the site of the rest area was developed. A specific failure plane was analyzed based on data obtained from the historical review and our field explorations. The failure plane slopes steeply to the east near the middle of Main Street, becomes planar for approximately 400 feet and slopes slightly up to the ground surface where it day-lights at the surface. The failure plane is parallel with the ground surface and located at a depth of 25 to 35 feet. The location of the cross section is shown on Quigley's map (Figure 1) with a few alterations to the ground surface to represent present day topography across the landslide area.

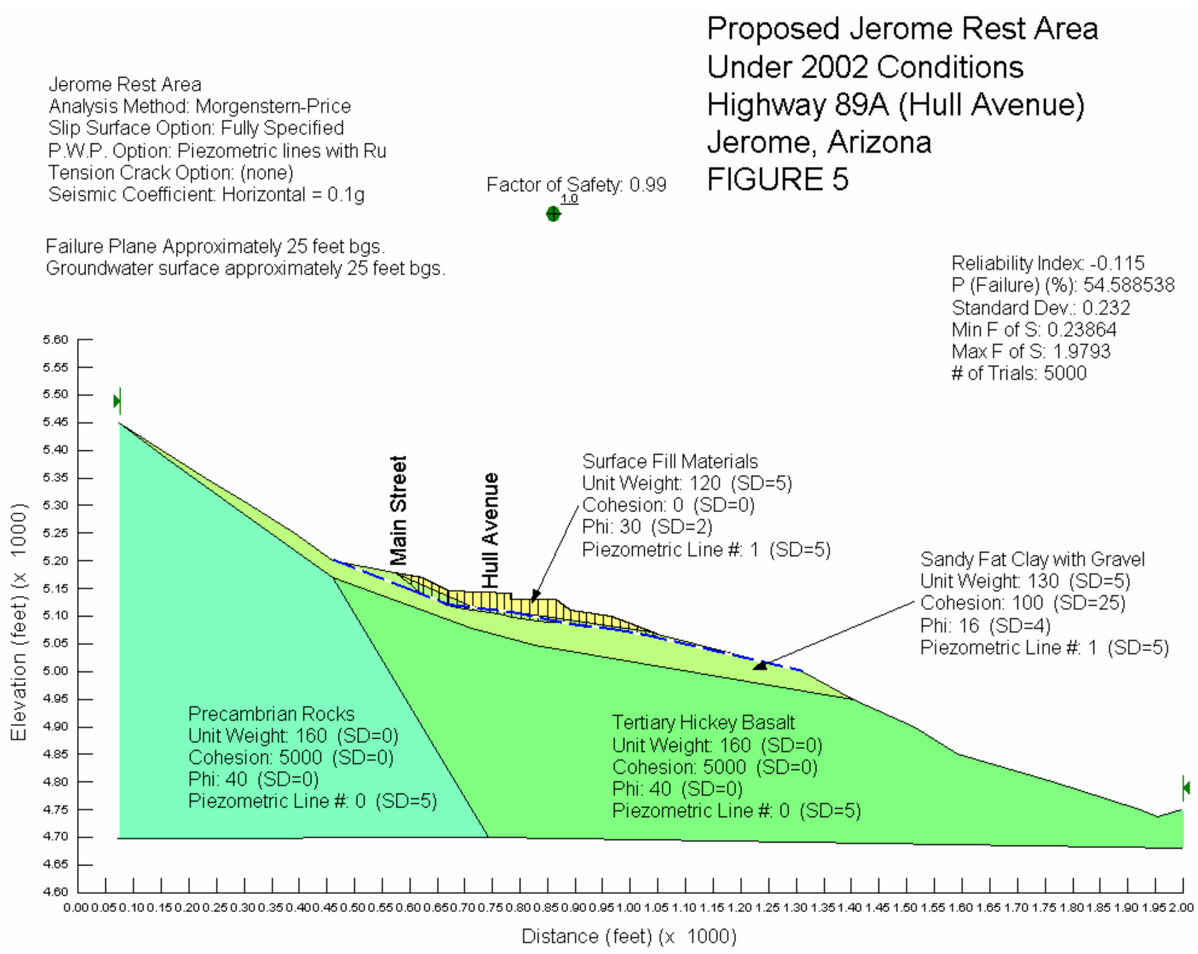

FIGURE 5: Slope Stability cross section; typical.

Slope stability analyses models were analyzed for 1936 conditions and 2002 conditions. The conditions included in the analyses are outlined below:

Parameter	1936	2002
Geometry	as shown on cross section	same as 1936
Failure Surface	specified	same as 1936
Depth to Groundwater	10 feet	25 feet
Friction Angle	16°	16°
Cohesion	0 psf	100 psf
Seismic Coefficient	none	varied

The seismic coefficient was varied under the 2002 conditions to ascertain the sensitivity of the analyses to the seismic coefficient. The cohesion under the 1936 conditions was reduced to zero considering the slide was in motion and the failure surface slightly adjusted until a factor of safety of 1.0 was achieved. The proximity of the factor of safety to 1.0 indicates the correlated residual friction angle, groundwater surface and failure surface assumptions are relatively close to those conditions that continued landslide movement.

Results of the stability analyses for each case and the corresponding calculated factors of safety are summarized in the following table.

Summary of Stability Analyses		
Condition Analyzed	**Seismic Coefficient**	**Factor of Safety**
1936	0.00g	1.0
2002	0.10g	1.0
	0.02g	1.3
	0.00g	1.5

Based upon our analyses, the slope is stable under 2002 conditions provided there are no strong ground motion forces added to the slope and based on current groundwater conditions. In addition, the results of the stability analyses indicate the safety factor is sensitive to the magnitude of the seismic coefficient.

Risk Analysis: The notion of risk is an important aspect of any geotechnical exploration. The primary reason for this is that investigative and analytical methods used to develop geotechnical conclusions and recommendations do not comprise an exact science. The analytical tools are generally empirical and must be tempered by engineering judgement and experience. The solutions or recommendations presented in any geotechnical study should not be considered risk-free and more importantly, are not a guarantee that the proposed structure will perform satisfactorily. What the engineering recommendations do constitute is the geotechnical engineers' best estimate of those measures that are necessary to make the structure perform satisfactorily based on usually limited subsurface information. The purpose of the following paragraphs is to discuss the concept of risk so the owner, who must ultimately decide what is an acceptable risk, can better apply the finding of this study.

As previously outlined, the most critical geotechnical consequence of this study is considered to be slope stability of the landslide area. The stability of a portion of this slope is expressed as a factor of safety. It is important to note the concept of factor of safety is a derived value and not an intrinsic property of the slope. The accuracy with which the factor of safety for a given slope can be determined, is based on a number of factors the most significant of which are listed below:

- Variability of surface conditions
- Variability and type of subsurface conditions
- Validity of the analytical method
- Validity of simplifying assumptions
- Intensity of study
- Certainty of the design loading conditions occurring.

Depending on how well the above factors can be assessed determines what minimum factor of safety would be required to have a reasonable degree of confidence that a failure will not occur. It is the geotechnical engineers' responsibility to assess these conditions and advise the owner as to a minimum acceptable factor of safety.

Theoretically, a factor of safety of 1.0 indicates that a slope is on the verge of instability. Therefore, any lower factor of safety should result in failure and any higher factor of safety should theoretically represent a safe slope. However, due to the uncertainties associated with any geotechnical investigation and the factors discussed in the preceding paragraph, all slopes, even those with factors of safety greater than 1.0, have some potential for failure. The higher the computed factor of safety is for a given slope, the lower its probability of failure will be. Approaches have been developed to relate computed factor of safety to probability of failure. This approach is called a probabilistic analysis and a limited risk analysis was performed for this study.

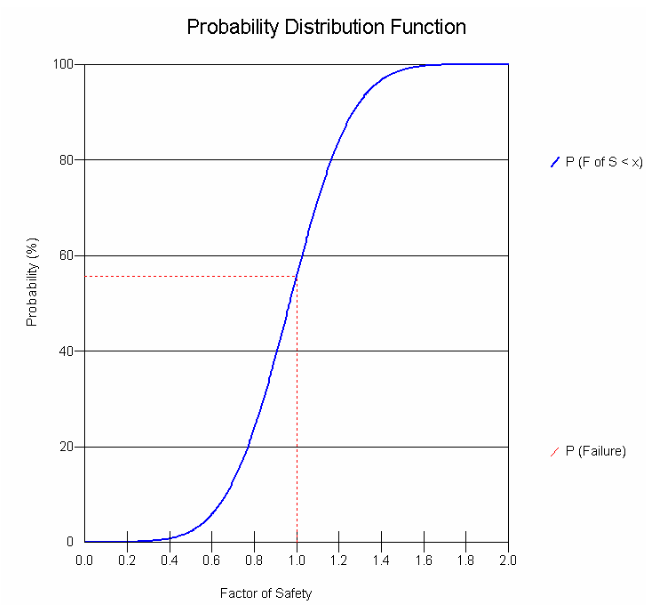

FIGURE 6: Probability Distribution graph for seismic coefficient of 0.1g.

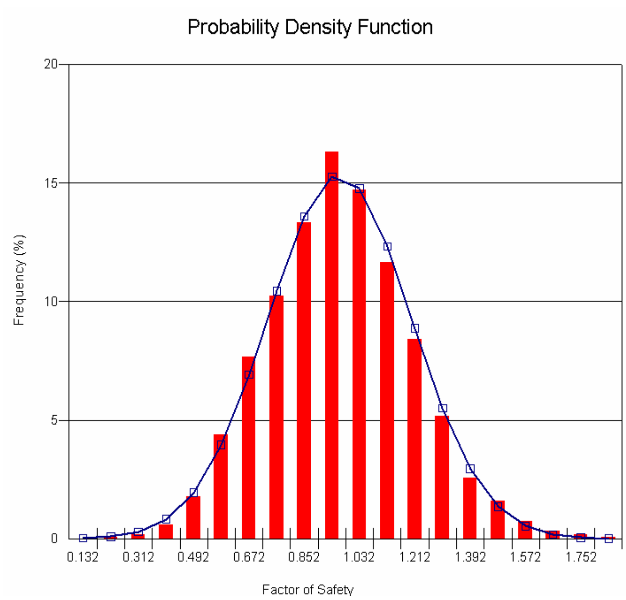

FIGURE 7: Probability Density graph for for seismic coefficient of 0.1g.

The list of parameters that were varied included unit weight, cohesion, friction angle, groundwater elevation, and seismic coefficient.

Conclusions: The results show there is generally a one in seven (15%) chance of slope instability under 2002 conditions when the seismic coefficient is 0.02g. When the seismic coefficient is 0.10g the probability is generally one in two (55%).

The risk of future landslide movement at the site is particularly sensitive to the seismic coefficient used in the slope stability analysis. Though the other parameters when varied do effect the slope stability, their effect is relatively small. When considering future development on this historic landslide, the forecasting of seismic or assimilated seismic events will be the most crucial parameter to acquire accurately.

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